ORIGINAL ARTICLE

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Displacement-based seismic design with damage control of asymmetric buildings

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https://doi.org/10.1590/1679-78256168

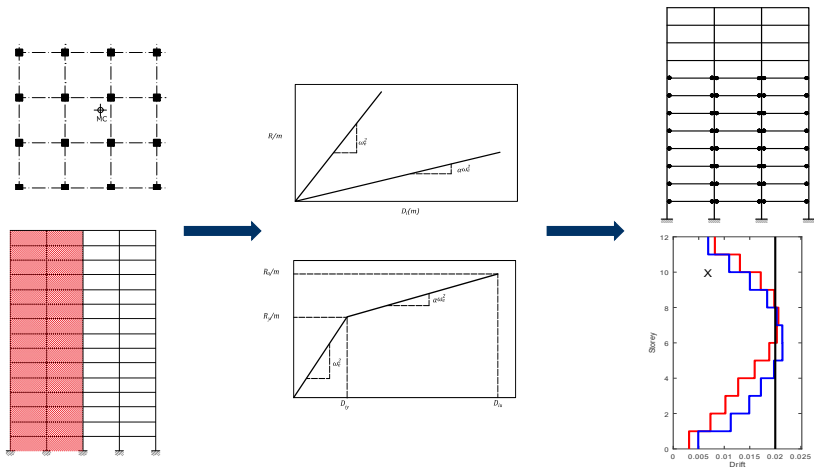
Abstract

This paper presents a procedure for the displacement-based seismic design of in-plan asymmetric buildings with earthquake-induced damage control. Damage is defined on the structural elements in the in-plan and elevation layout of the structure. The proposed method is based on the concepts of the performance-based seismic design philosophy and the application of basic equations of structural dynamics that are regularly used for the current design of buildings. In its application, the simultaneous bidirectional seismic demand is characterized by smooth design spectrum as proposed by most current regulations. To illustrate the steps required in the application of the design method proposed, the paper presents the design process of a 15- and 12-story buildings with in-plan asymmetric distribution of stiffnesses and subjected to a design demand given by the spectrum a real seismic event representative of soft soil sites such as those of the bed-lake of Mexico City. The results obtained are compared with the corresponding results of the nonlinear dynamic step-by-step analyses under the same seismic demand. Based on the results obtained, the relevance of this procedure in the displacement-based seismic design of asymmetric buildings and the implications for its consideration in future regulations are discussed.

Keywords

Displacement-based seismic design; asymmetric buildings; damage control; modal spectral analysis

Graphical Abstract



Received: June 26, 2020. In Revised Form: September 23, 2020. Accepted: October 26, 2020. Available online: November 06, 2020.
https://doi.org/10.1590/1679-78256168

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1 INTRODUCTION

For the design of buildings for limit states involving structural damage, most seismic codes accept the use of a force-based procedures, using, with certain restrictions, the static method or, without restrictions, dynamic methods such as modal spectral analysis, regardless of the fact that the seismic effects on structures are better represented by displacement-based design procedures. Particularly for asymmetric buildings, force-based methods assume a linear viscoelastic behavior, even for limit states involving seismically induced structural damage explicitly considering the effects of torsion through eccentricities corresponding to undamaged conditions. In this design option, the existence of in-plan asymmetries in a structure is penalized by reductions of the seismic behavior factor used for its design. However, several studies have shown the limitations of this approach, since the distribution of forces in in-plan asymmetric buildings in the non-linear range of behavior may differ significantly from that in the elastic range. Moreover, the aforementioned scheme of penalizing seismic demands leads to a uniform alteration of the design strengths of all structural elements, so it is not particularly effective to reproduce the damage states and collapse mechanism occurring in the designed structure when subjected to the design demands, an issue that is essential to guarantee a target seismic performance under design conditions.

Of particular relevance to the seismic design of in-plan asymmetric buildings are the results of investigations of diverse authors stressing the limitations of the force-based design procedures. Accordingly, various authors have conducted research on the influence of torsional effects on the overall seismic performance of structures, proposing to reduce these effects by suitable control of the distribution of stiffnesses and strengths. Paulay (2002) issued recommendations to consider explicitly the effect of strength distributions in determining the nonlinear torsional behavior of in-plan asymmetric buildings and discussed their implications when used in the performance-based seismic design of such structures. Unfortunately, these investigations have been only partly continued, e.g., Castillo (2004) and Beyer (2007), making evident the need to intensify them, given the importance of the results expected from them.

Based on the assumption that an approximation to the nonlinear performance of a multiple degrees of freedom (MDOF) structure may be obtained from the performance of a reference single degree of freedom (SDOF) structure, generally associated with the fundamental mode of the buildings and particularly with the recognition of the advantages of a displacement-based design approach over the conventional force-based design procedures recommended by most seismic design codes for building structures, in this paper, the authors propose a new displacement-based seismic design method that guarantees the target performances, allowing for an appropriate control of earthquake-induced damage of in-plan asymmetric buildings. In this method, seismic design forces are directly calculated from a design displacement profile, associated with an ultimate limit state. This profile is defined from a target interstory drift, a distribution of damage under design conditions and the physical layout of the considered irregular building, hence allowing explicit damage control of the structure under design conditions. In the application of the design, a method proposing simultaneous bidirectional seismic demand characterized by smooth design spectra, as recommended by the current regulations, is used.

To illustrate the steps involved in the practical implementation and application of the method proposed, the design of two 12- and 15-story reinforced concrete buildings with in-plan asymmetric mass and stiffness distributions and subjected to a bidirectional seismic demand characteristic of soft soil sites in Mexico City is presented. The results obtained are compared with the corresponding results obtained from nonlinear step-by-step analyses, showing an acceptable correspondence with the interstory drift used as a target design performance index.

2 BACKGROUND

Lucchini et al. (2011) studied the torsional response of one-level asymmetrical buildings under biaxial demand, considering several angles of incidence. They found that the parameters governing the nonlinear floor response of asymmetric buildings are those associated with the system's resistance center and correspond to the basal shear produced by the inelastic mechanisms in each direction.

Lumantarna et al. (2011) mention the deficiencies of the conventional force-based method and point out that a seismic performance of this nature can be explained by what is known as the displacement-controlled phenomenon. The assumption of a peak displacement demand can potentially simplify the seismic design or assessment of a structure which is flexible or has the capacity to undergo large displacement without collapsing. A generalized response spectrum model which features a displacement-controlled phenomenon was first developed to provide seismic response predictions assumed as linear.

Miranda et al. (2012) They found that an increase in the strength of an element does not increase the demand on any critical element. An increase in rotational mass or a decrease in stiffness eccentricity reduces the critical wall displacement.

Hong (2013) studied the torsional response for one-way or two-way asymmetric systems under uni- and bidirectional seismic excitations. He considered instantaneous load eccentricities caused by the motion of the center of mass. They found that the eccentricities, which are time-dependent, exist even for linear elastic two-way symmetric structures under seismic excitations, as the relative displacement between the center of mass and center of stiffness may not necessarily be negligible.

Lumantarna et al. (2013) mention that the displacement-controlled behavior is a feature of low to moderate seismicity areas where the peak displacement demand on structures could be limited despite significant structural strength and stiffness degradation. They studied the displacement-controlled phenomenon in torsionally unbalanced framing systems and concluded that while torsional actions are a well-researched topic, the incorporation of a displacement-controlled phenomenon in the analysis is original and represents a new development.

Georgoussis (2013) investigated the problem of strength distribution among the resisting elements and the yield shear of a single-story inelastic structure with simple eccentricity, with respect to minimizing its torsional response during a ground motion. The aim was to present an interaction relationship between the yield shear and the maximum torque that may be developed in such systems, and also to examine the response of such model structures under characteristic ground motions.

Gokdemir et al. (2013) analyzed the effects of irregularity on structures. Building models, which have different numbers of floors and floor areas, are generated by a computer program. Their results compared the prevention of damage caused by torsional irregularity under earthquake loads, as well as statements in different earthquake codes about torsional irregularity. They showed that separating big building sections from each other with proper separation distances and increasing the lateral rigidity in the weak direction of the structures both decrease the effect of torsion.

Wakchaure and Nagare (2013) mention that the torsional behavior of asymmetric buildings is one of the most frequent causes of structural damage and failure during strong ground motions. They studied the influence of the torsion effects on the behavior of the structures, investigating two cases in the building, with and without considering torsion.

Georgoussis (2015) analyzed the common types of multistory buildings, detailed according to a planar static analysis under a code lateral loading, which may have a practically translational behavior when the mass axis is passing through the modal center of rigidity. They specify the optimum arrangement of these structural elements in terms of producing the minimum torsional response in the case of a strong ground motion. This is demonstrated in typical five-story buildings, composed of moment-resisting frames and structural walls, under the effect of ground motions.

Fox et al. (2016) investigated the seismic response of single-story asymmetric-plan systems subjected to unidirectional earthquake excitation. Their work develops existing displacement-based assessment principles and has a strong focus on the mechanics of the problem at hand. The procedure relies on the assumption that the center-of-mass displacement can be accurately determined using an equivalent single-degree-of-freedom system. The rotation of the system is then determined using a novel approach that assigns effective stiffness properties to the structural elements.

Fakhraddini et al. (2019) studied a group of 30 eccentrically braced frames under a set of 15 far-field and near-field accelerograms, which they scaled to different amplitudes to adapt various performance levels. The results were post-processed by nonlinear regression analysis in order to recognize the major parameters that influence the peak displacement pattern of these frames.

However, in the last 20 years great efforts have been made to develop new performance-based methodologies, particularly in displacement, currently there is still no validated methodology that considers the control of the distribution of resistance in the non-linear range. According to the studies by Paulay (2002), where it was demonstrated that when there is adequate resistance control, it can be achieved that a structure with multiple degrees of freedom can be idealized as an oscillator of one degree of freedom, allowing the approximate methods are applicable. In this work, we consider the control of the distribution of resistances by means of parameters obtained from the design codes associated with the local damage of the structural elements.

3 ESTIMATION OF THE TARGET DESIGN DISPLACEMENT FROM MODAL SPECTRAL ANALYSIS

For most practical seismic design applications, the current codes recommend the use of modal spectral analyses to estimate the maximum values of the selected design performance indexes. In the case of the design of building structures subjected to bidirectional seismic demands, both characterized by a single design spectrum, the times at which the corresponding maximum performance index occurs are not known, making it necessary to use a code-prescribed

combination rule to approximate the maximum performance index associated with both bidirectional demands acting simultaneously. To avoid the implication of this approximation, in this paper the foundations of the design method proposed will be initially presented as if it were carried out in the time domain.

Based on the motion equation of a degree of freedom oscillator (Paz and Leigh, 2004) subjected to a bidirectional seismic action we define the decoupled dynamic equilibrium equation, which in terms of modal displacements is as follows.

$$\ddot{v}_i + 2\zeta_i\omega_i\dot{v}_i + \omega_i^2 v_i = -\Gamma_{xi}\ddot{u}_{gx}(t) - \Gamma_{yi}\ddot{u}_{gy}(t) \tag{1}$$

where $2\zeta_i\omega_i$, ω_i^2, Γ_{xi} and Γ_{yi} are the modal damping, the modal frequencies and the participation factors respectively.

Decomposing v_i, $\dot{v}_i$ and $\ddot{v}_i$ into the terms of two components, each corresponding to each of the bidirectional seismic demand terms in Eq. (1),

$$v_i = v_{ix} + v_{iy} \tag{2}$$

Using Eq. (1), Eq. (2) may then be separated into two uncoupled dynamic equilibrium equations:

$$\ddot{v}_{ix} + 2\zeta_i\omega_i\dot{v}_{ix} + \omega_i^2 v_{ix} = -\Gamma_{xi}\ddot{u}_{gx}(t) \tag{3}$$

$$\ddot{v}_{iy} + 2\zeta_i\omega_i\dot{v}_{iy} + \omega_i^2 v_{iy} = -\Gamma_{yi}\ddot{u}_{gy}(t) \tag{4}$$

The dynamic equilibrium equations for the two oscillators, unit mass, and single degree of freedom (SDOF), each subjected to the respective $\ddot{u}_{gx}(t)$ and $\ddot{u}_{gy}(t)$ demands, are

$$\ddot{D}_{ix} + 2\zeta_i\omega_i\dot{D}_{ix} + \omega_i^2 D_{ix} = -\ddot{u}_{gx}(t) \tag{5}$$

$$\ddot{D}_{iy} + 2\zeta_i\omega_i\dot{D}_{iy} + \omega_i^2 D_{iy} = -\ddot{u}_{gy}(t) \tag{6}$$

where D_{ix} and D_{iy} are the respective displacements produced by the demands $\ddot{u}_{gx}(t)$ and $\ddot{u}_{gy}(t)$. Comparing, one to one, Eqs. (3)–(4) with Eqs. (5)–(6),

$$v_{ix}(t) = \Gamma_{xi}D_{ix}(t) \tag{7}$$

$$v_{iy}(t) = \Gamma_{yi}D_{iy}(t) \tag{8}$$

Substituting Eqs. (7)–(8) into Eq. (2),

$$v_i(t) = \Gamma_{xi}D_{ix}(t) + \Gamma_{yi}D_{iy}(t) \tag{9}$$

The displacements for the building associated with mode i^{th} may be written as

$$u_{ix}(t) = \phi_{ix}\left(\Gamma_{xi}D_{ix}(t) + \Gamma_{yi}D_{iy}(t)\right) \tag{10}$$

$$u_{iy}(t) = \phi_{iy}\left(\Gamma_{xi}D_{ix}(t) + \Gamma_{yi}D_{iy}(t)\right) \tag{11}$$

$$u_{i\theta}(t) = \phi_{i\theta}\left(\Gamma_{xi}D_{ix}(t) + \Gamma_{yi}D_{iy}(t)\right) \tag{12}$$

Alternatively, as the maximum values of D_{ix} and D_{iy}, $\overline{D}_{ix}$ and $\overline{D}_{iy}$, occur at different times, these values may be combined to calculate the maximum displacement of mode i^{th} under both demands acting simultaneously, as it really occurs, *i.e.,* Δ_{irx} and Δ_{iry} (roof displacement of i^{th} mode in x and y respectively) may be calculated using a factor between the responses to these demands, β, *e.g.,* if the maximum displacement occurs in the X direction, *i.e.,* Δ_{irx},

$$\Delta_{irx} = \phi_{irx}\Gamma_{xi}\overline{D}_{ix} + \beta\phi_{irx}\Gamma_{yi}\overline{D}_{iy} \tag{13}$$

and

$$\overline{D}_{ix} = \frac{\Delta_{irx} - \beta\phi_{irx}\Gamma_{yi}\overline{D}_{iy}}{\phi_{irx}\Gamma_{xi}} \tag{14}$$

4 DISPLACEMENT-BASED SEISMIC DESIGN METHOD WITH DAMAGE CONTROL

The displacement-based seismic design method proposed in this paper relies on the assumption that it is possible to approximate the maximum seismic response of an MDOF structure via the response of a reference SDOF oscillator with dynamic characteristics consistent with those corresponding to the fundamental mode of vibration of the structure.

Under the aforementioned assumption, the dynamic capacity curve of an MDOF structure may be approximated by a bilinear curve (Figure 1a) and, from such, the spectral displacement *vs.* spectral pseudo-acceleration (strength per unit mass) curve of the reference SDOF system (Figure 1b). This curve, called the behavior curve in the framework of the method, is used to approximate the maximum response of the MDOF structure. The slope of the first branch of this behavior curve represents the stiffness of the reference SDOF system in the elastic range, *i.e.,* the undamaged structure, and the slope of the second branch, the stiffness in the post-yield range of behavior, *i.e.,* the structure with the distribution of damage, expected under design conditions, which is proposed by the designer. The strength per unit mass $\left(R_y/m\right)$, corresponding to the crossing of the two branches of the behavior curve, represents the intensity of seismic demand for which the structure will first exhibit the damage distribution when subjected to the design demands.

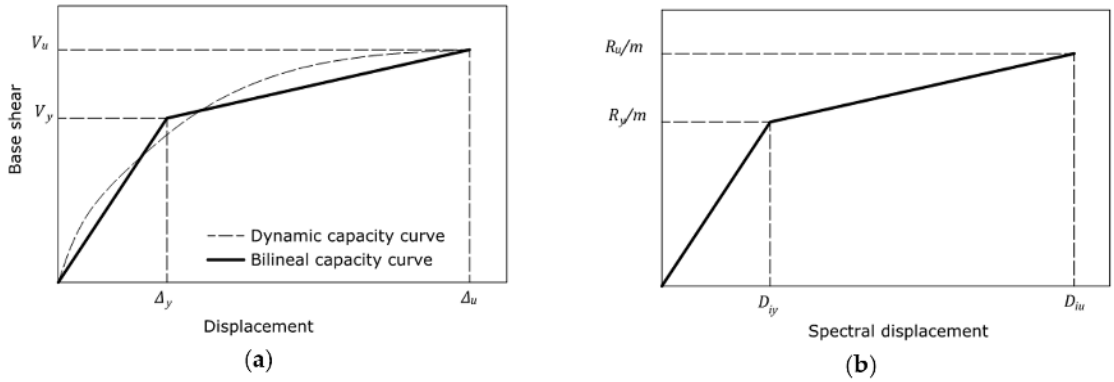


Figure 1 Illustration of the capacity curve of a structure and the behavior curve of the corresponding reference SDOF system as those used by the design method proposed: (a) Capacity curve, real and its bilinear approximation, (b) Approximate behavior curve in the ADRS format.

The ultimate strength per unit mass $\left(R_u/m\right)$ represents the intensity of seismic demand for which the structural elements assumed to remain undamaged under design demands do so. Having defined the properties of the reference SDOF system, modal spectral analyses are used to estimate the maximum performance under bidirectional seismic demands applied separately in each orthogonal direction. A code-defined factor to combine the maximum performances

under these demands acting separately, β, is used, *e.g.,* the Mexico City Building code recommends the use of a 30% combination rule or the SRSS rule.

4.1 Determination of the target maximum displacement of the reference SDOF system.A sub-section

This displacement is defined in terms of the maximum allowable value of interstory drift associated with the design performance level:

$$\delta_d = \delta_{d1} + \delta_{d2} \tag{15}$$

where δ_{d1} and δ_{d2} are the design interstory drifts.

For i^{th} mode contributions, δ_{di1} and δ_{di2} are calculated using Eqs. (16) and (17). The subscripts 1 and 2 correspond to the elastic and the post-yield behavior stages

$$\delta_{di1} = \rho_{i1}\delta_{d1} \tag{16}$$

$$\delta_{di2} = \rho_{i2}\delta_{d2} \tag{17}$$

where ρ_{i1} and ρ_{i2} are the ratio between the interstory drifts calculated considering only the reference mode i^{th} and the ratio considering all modes, as shown in Eqs. (18) and (19).

$$\rho_{i1} = \frac{\delta_{i1}}{\sum_i^n \delta_{i1}} \tag{18}$$

$$\rho_{i2} = \frac{\delta_{i2}}{\sum_i^n \delta_{i2}} \tag{19}$$

where δ_{i1} and δ_{i2} interstory drifts of i^{th} modal form. To calculate the displacements corresponding to the mode with the highest contribution to the response corresponding to each of the performance stages, *i.e.,* Δ_{ir1} and Δ_{ir2}, the following equations are used:

$$\Delta_{ir1} = \frac{\delta_{di1}}{\delta_{i1}} \tag{20}$$

$$\Delta_{ir2} = \frac{\delta_{di2}}{\delta_{i2}} \tag{21}$$

The maximum displacements of the reference mode for each stage are calculated with the following equations as $\overline{D}_{ix}$ for x projection:

$$\overline{D}_{ix1} = \frac{\Delta_{ir1} - \beta\phi_{irx}\Gamma_{yi}\overline{D}_{iy1}}{\phi_{irx}\Gamma_{xi}} \tag{22}$$

$$\bar{D}_{ix2} = \frac{\Delta_{ir2} - \beta\phi_{irx}\Gamma_{yi}\bar{D}_{iy2}}{\phi_{irx}\Gamma_{xi}} \tag{23}$$

4.2 Proposed methodology

Based on the above fundamentals, the detailed steps required for the application of the method proposed for the displacement-based design of buildings with in-plan asymmetries and subjected to bidirectional seismic demands can be summarized in what follows:

- Construction of the structural model of the pre-designed undamaged building. The pre-design of the building for the considered demands may be obtained in accordance with current force-based design regulations, under gravity and seismic loads, either through a static analysis or by modal spectral analysis.
- Definition of the performance index associated with the performance objective. The target performance index used for the displacement-based design of buildings is, generally, the maximum interstory drift prescribed by a building code.
- Definition of an acceptable configuration of structural damage under design conditions. This configuration is defined by selecting the structural members and sections in which damage under the design conditions is accepted to occur. Damage is introduced into the structural model, here referred to as "damaged", in accordance with the characteristics of the model used to carry out the seismic analyses of the building, e.g., plastic hinges at the ends of a damaged element.
- Determination of the dynamic characteristics of the undamaged and damaged structural models. Eigenvalue analysis of both models with the aim of obtaining modal periods and shapes is carried out. From the results of these analyses, the dynamic characteristics of the mode that most influences the response of the structure are defined, i.e., the frequencies associated with the branches of the behavior curve of the reference SDOF; one frequency for the elastic, undamaged, branch, ω_e, and the other for the inelastic, damaged, branch, $\alpha\omega_e$ (Figure 2).

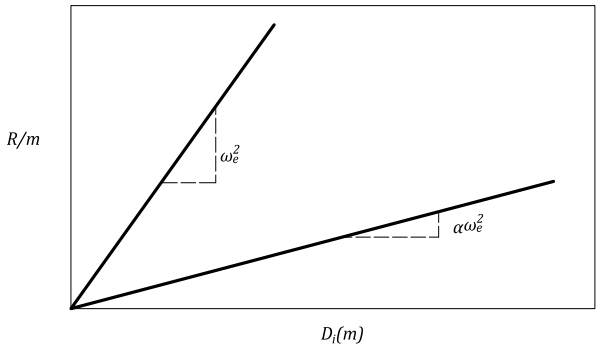


Figure 2 Branches of the behavior curve of the reference SDOF system.

- Calculation of the yield displacement of the reference SDOF system. The displacement $\bar{D}_{i1}$ in the behavior curve is defined in terms of the yield interstory drift, where the maximum drift occurs in the structure (Priestley and Kowalsky , 2000, Lopez 2009).
- Determination of the target maximum displacement of the reference SDOF system (see section 3.1).
- Calculation of the displacement of the reference SDOF system. From an inelastic displacement spectrum, associated with ductility, μ, calculated in the previous step and the value of α defined in Figure 3, the spectral interstory displacement $\left(D_{ux}\right)$ corresponding to the fundamental period of the elastic model, T_1, is obtained (Figure 4). This displacement is compared with the target displacement $\left(\bar{D}_{ix2}\right)$; if they are equal, the design process continues; if not, the initial structure and/or the distribution of damage proposed is modified and the analysis is reinitiated. Depending on what is modified, all this is carried out in an iterative fashion until the closeness of such displacements is reached.

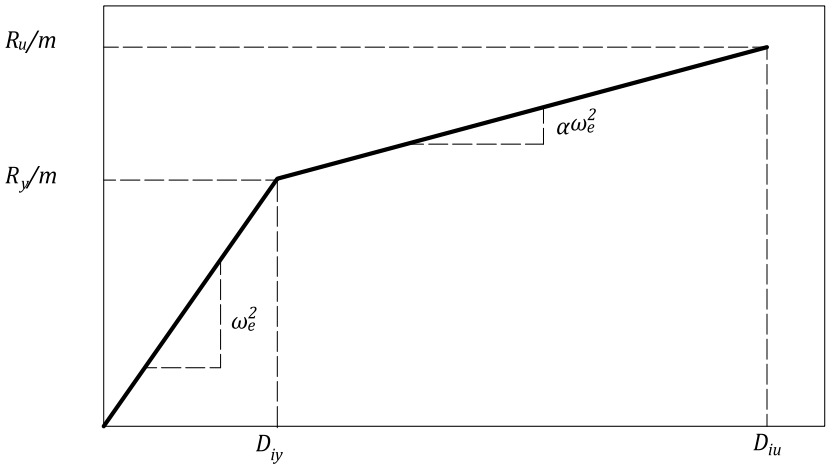


Figure 3 Behavior curve of a reference SDOF system.

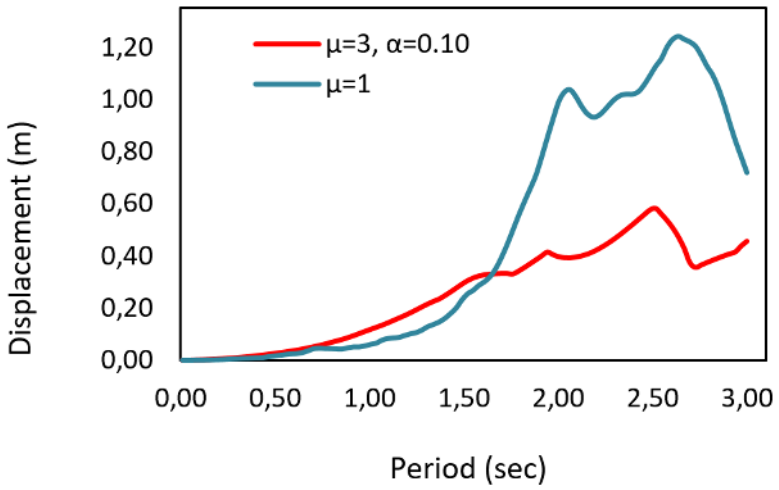


Figure 4 Elastic and inelastic displacement spectra.

Calculation of the design forces. For the calculation of the design forces of the structural elements, including the contribution of the gravitational and other permanent loads, two modal spectral analyses, here carried out in the time domain for validation purposes, need to be carried out. One analysis is for the undamaged model, using as seismic demands the records scaled by the λ_1 factor defined as the ratio of the yield displacement of the oscillator to the maximum elastic displacement (Eq. (24)). The other analysis is for the model with damage included, using as seismic demand the same records, scaled by the λ_2 factor defined as the ratio between the target displacement and maximum displacement corresponding to stage 2 (Eq. (25)). The design forces for the structural elements are obtained by adding the forces of the two modal spectral analyses.

$$\lambda_1 = \frac{\bar{D}_{i1x}}{\bar{D}_{1x}} \tag{24}$$

$$\lambda_2 = \frac{\bar{D}_{i2x}}{\bar{D}_{2x}} \tag{25}$$

The proposed methodology is summarized in the flowchart in Figure 5.

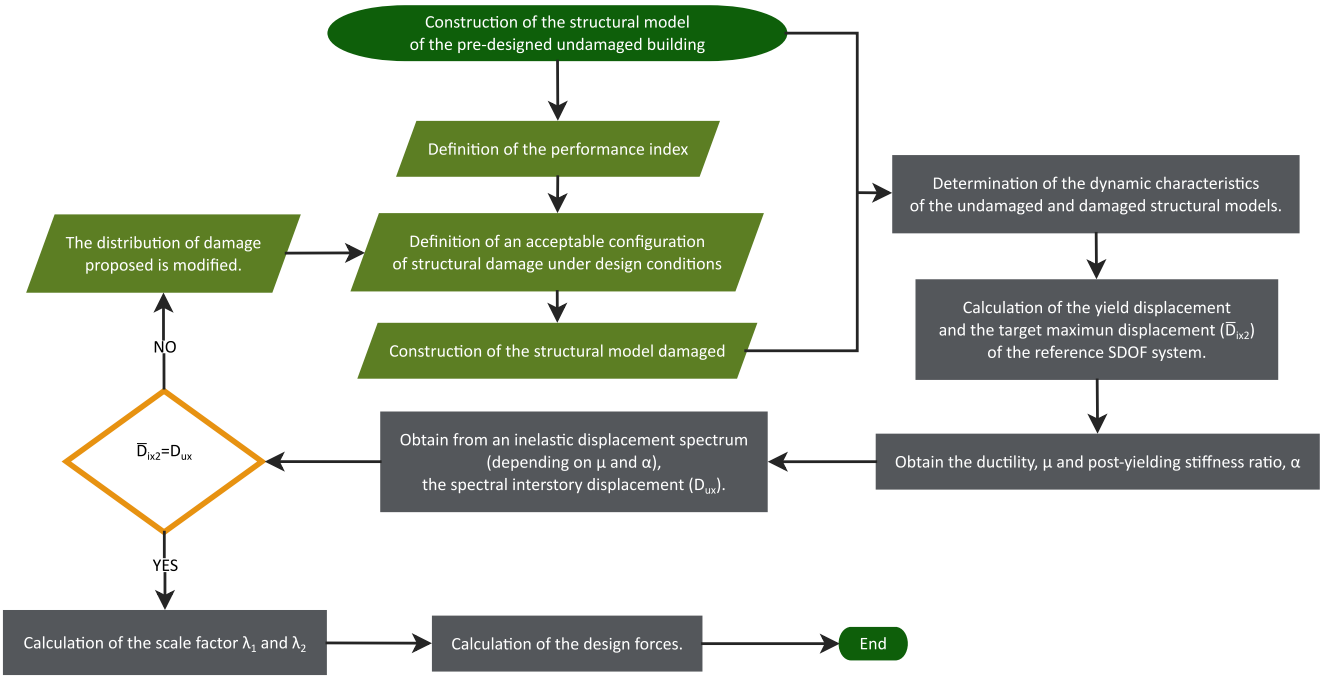


Figure 5 Flowchart of the displacement-based seismic design method with damage control.

5 APPLICATION OF THE DESIGN METHOD PROPOSED

5.1 Description of the buildings used for illustration.

To validate and illustrate the application of the design method proposed, two reinforced concrete in-plan asymmetric buildings, 15- and 12-story high, were used. The 15-story building has a rectangular plan with three bays 7m long in the X direction and four bays 8m long in the Y direction. Reinforced concrete frames with walls 0.2m thick form the X1 and Y1 axes of the building. The slabs are 0.12m thick and the interstory spacing is 3.3m high. Figure 6 illustrates the building plan and the frames with walls in both orthogonal directions.

The 12-story building also has a rectangular plan with three bays 10 m long in the X and Y directions. The slabs are 0.12m thick and the interstory spacing is 3m. The building has a 10% eccentricity of design. Figure 7 illustrates the building plan and the frames in both orthogonal directions.

The properties of the materials were, for concrete: strength: $f'c = 24.52Mpa$, Young's modulus $E_c = 21707.90Mpa$ and weight density $W_D = 23.54\,kN/m^3$, and for steel: $fy = 411.88Mpa$.

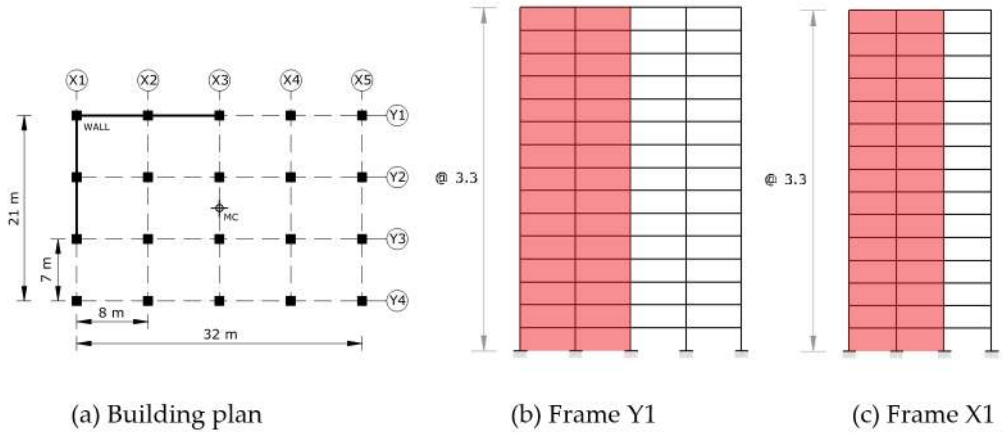


Figure 6 15-story building.

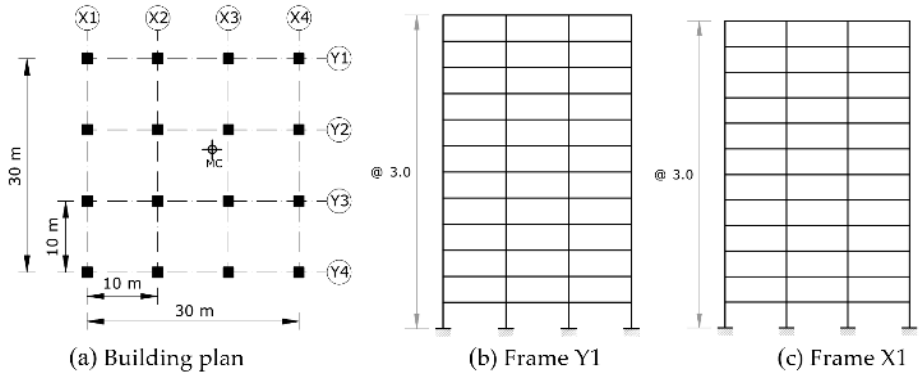


Figure 7 12-story building.

5.2 Design considerations.

For these examples, the performance index used as the target was a 0.02 maximum interstory drift as recommended for the ultimate limit state by the Mexico City Building Code and its Complementary Technical Standards (CDMX 2017). Both buildings were designed using as seismic demand the acceleration records of the horizontal components of a real earthquake so as to be able to validate the method by comparing the expected performances of the buildings with those obtained from the nonlinear dynamic step-by-step analyses of these buildings subjected to the same seismic demand used for their design. In this paper, the time history of displacements under both demand components acting simultaneously is obtained by summing the time histories of the displacements calculated separately for each demand component.

For the design of each building, the following considerations were taken:

- The behavior curve of the reference SDOF system is constructed using the parameters obtained using the steps described in section 3, *i.e.,* yield displacement, ultimate displacement, yield strength, ultimate strength, initial stiffness, post-yield stiffness, ductility and initial to post-yield stiffness ratio (Figure 3).
- For damaged models are the beam-hinges with zero rotational stiffness considered.
- No damage is considered to occur at the walls, at the base of the columns of the first level, or on the beams of the 4 upper levels.

5.3 Design seismic demands.

For the design of the buildings, the records of the NS and EW components of the September 19, 1985 earthquake obtained at the SCT station in Mexico City were used as design demands. The reference SDOF system used in the application of the design method corresponded to the fundamental mode of the structures. Modal time history analyses were performed with the commercial structural analysis program SAP2000 (CSI 2009) and the dynamic nonlinear step-by-step analyses with the PERFORM 3D V5 program (CSI 2011).

Although, the proposed method is designed to consider smoothed design spectra as seismic demand, for validation purposes, the application examples will use modal analyses over time, preventing some non-methodological parameters from affecting the resulatstodo, such as different combination rules. Two orthogonal records are considered as seismic demand, 100% in each direction.

5.4 Analysis of results of the example building designs.

To demonstrate the validity and scope of the design method proposed, the performance objective and other results of the method are compared with those calculated with a nonlinear dynamic step-by-step analysis under the same design demands.

The design forces of the structural elements are calculated using two modal analyses in time, the first for the undamaged structure ($\lambda_1 = 0.24$ for the 15-story building and $\lambda_1 = 0.50$ for the 12-story) and the second for the damaged structure (factor, $\lambda_2 = 0.60$ for the 15-story building and $\lambda_2 = 0.70$ for the 12-story). Table 1 shows the properties of the behavior curve of these structures.

Table 1 Properties of the behavior curve.

Property	12-story	15-story
Yield displacement $(\bar{D}_{i1})$	0.13 m	0.15 m
Strength per unit mass at yield (S_{ay})	1.55 m/s^2	1.79 m/s^2
Ultimate displacement $(\bar{D}_{ix2})$	0.28 m	0.30 m
Ultimate strength per unit mass (S_{au})	1.75 m/s^2	1.86 m/s^2
Ratio of post-yield to elastic stiffness (α)	7.5%	10%
Ductility (μ)	2.1	2.0

Tables 2 to 5 they show the modal parameters of the models with damage and without damage of the buildings analyzed. The approximate static eccentricity of the 15-storey building is 35% in the X direction and 44% in the Y direction; for the 12-storey building the static eccentricity for both orthogonal axes is 10%. The torsional stiffness ratio for 15-storey building is 7% and for 12-storey is 50%.

Table 2 Modal parameters for undamaged 15-story building.

Mode	Period (T)	Modal mass ratio		
		X	Y	RZ
1	1.8452	0.1783	0.4199	0.1733
2	1.1584	0.4661	0.2427	0.0060
3	0.5940	0.0130	0.0400	0.0578
4	0.4992	0.1077	0.0981	0.5378
5	0.3277	0.0077	0.0196	0.0131
6	0.3177	0.1082	0.0631	0.0028

Table 3 Modal parameters for damaged 15-story building.

Mode	Period (T)	Modal mass ratio		
		X	Y	RZ
1	5.8179	0.1652	0.3858	0.1811
2	1.3932	0.4525	0.2433	0.0061
3	1.0614	0.0274	0.0656	0.0335
4	0.5265	0.1120	0.0977	0.5279
5	0.4391	0.0042	0.0150	0.0342
6	0.3382	0.1145	0.0674	0.0032

Table 4 Modal parameters for undamaged 12-story building.

Mode	Period (T)	Modal mass ratio		
		X	Y	RZ
1	1.6633	0.3231	0.3231	0.0872
2	1.5540	0.3665	0.3665	0.0000
3	1.1146	0.0433	0.0433	0.6479
4	0.5614	0.0560	0.0560	0.0149
5	0.5239	0.0637	0.0637	0.0000
6	0.3771	0.0077	0.0077	0.1106

Table 5 Modal parameters for damaged 12-story building.

Mode	Period (T)	Modal mass ratio		
		X	Y	RZ
1	6.0753	0.2955	0.2955	0.0841
2	5.6645	0.3376	0.3376	0.0000
3	4.0899	0.0421	0.0421	0.5912
4	1.2048	0.0680	0.0680	0.0193
5	1.1233	0.0777	0.0777	0.0000
6	0.8110	0.0097	0.0097	0.1360

Figures 8 and 9 show a comparison between the damage distributions in frames X5 for the 15-story building and X4 for the 12-story building used for the application of the design method proposed and that attained from the nonlinear dynamic step-by-step analysis. It is noted that the frames exhibited damage distributions similar to those proposed, so it may be concluded that the damage control for both buildings was satisfactory. In general, the proposed damaged distributions are maintained, keeping the columns and wall in an elastic state, as it was estimated in the seismic design.

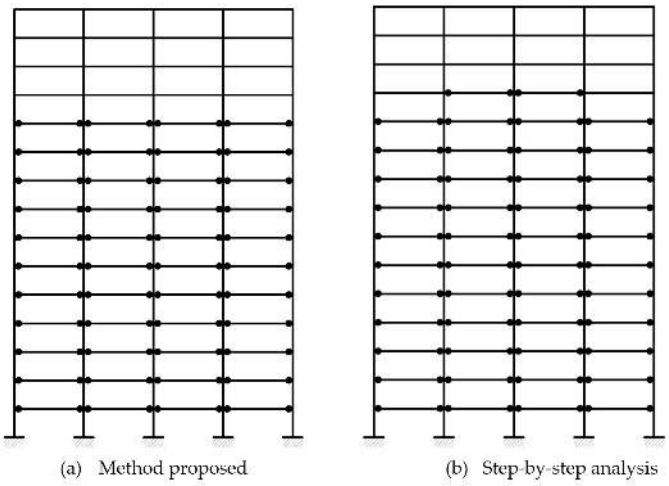


Figure 8 Damage distribution in frame X5 of the 15-story building.

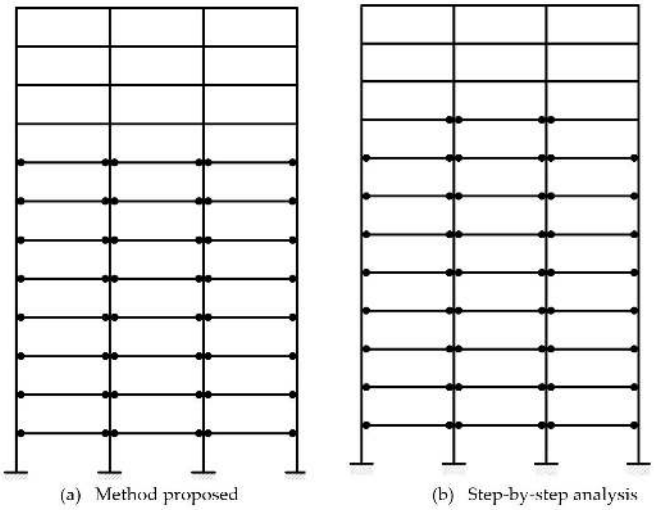


Figure 9 Damage distribution in frame X4 of the 12-story building.

Figures 10 to 13 show the comparisons between the displacements profile and interstory drifts obtained with the proposed method and those calculated via the nonlinear step-by-step dynamic analyses along the intersections of frames

X5-Y1, X1-Y4 for the 15-story building and X4-Y1, X1-Y4 for the 12-story building respectively. It is noted that the maximum interstory drifts in the Y direction in the 15-story building and in the X direction in the 12-story building are similar to those obtained using nonlinear step-by-step analysis. However, these figures also show that the displacement profiles in the X and Y directions for both buildings obtained with the method proposed and the corresponding profiles calculated through the step-by-step analyses are not necessarily as close as those corresponding to the interstory drifts.

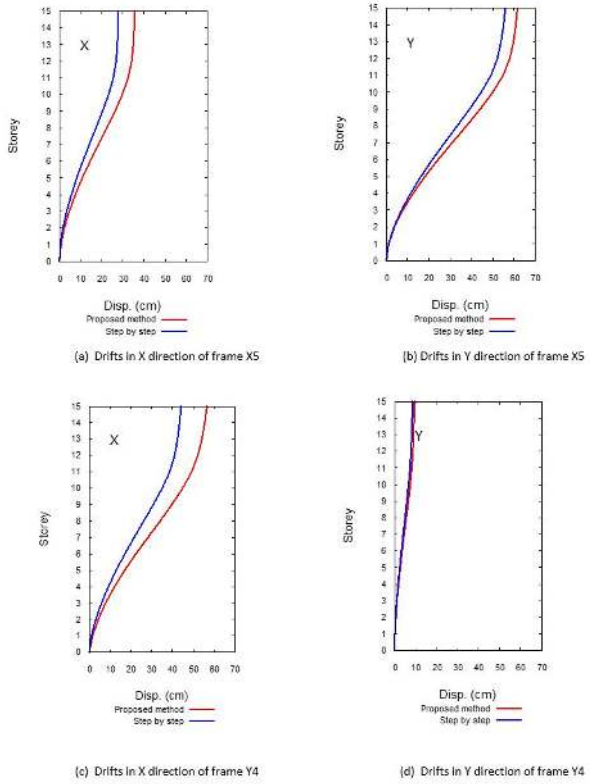


Figure 10 Comparison of the profile of maximum horizontal displacements for15-story building.

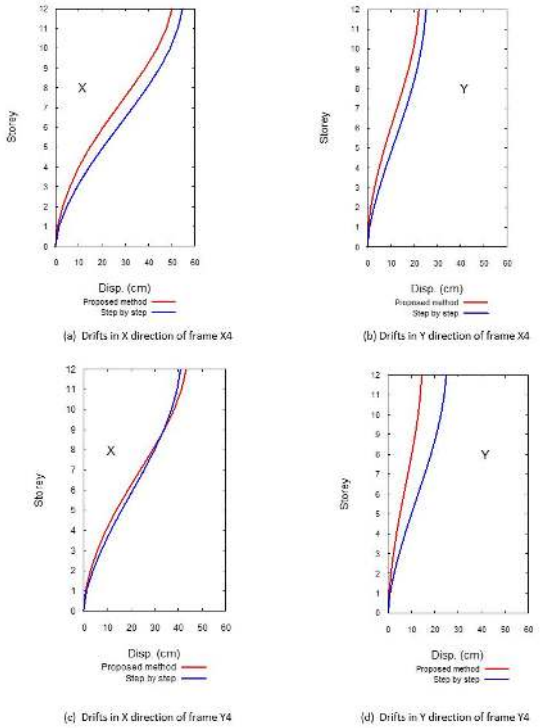


Figure 11 Comparison of the profile of maximum horizontal displacements for12-story building.

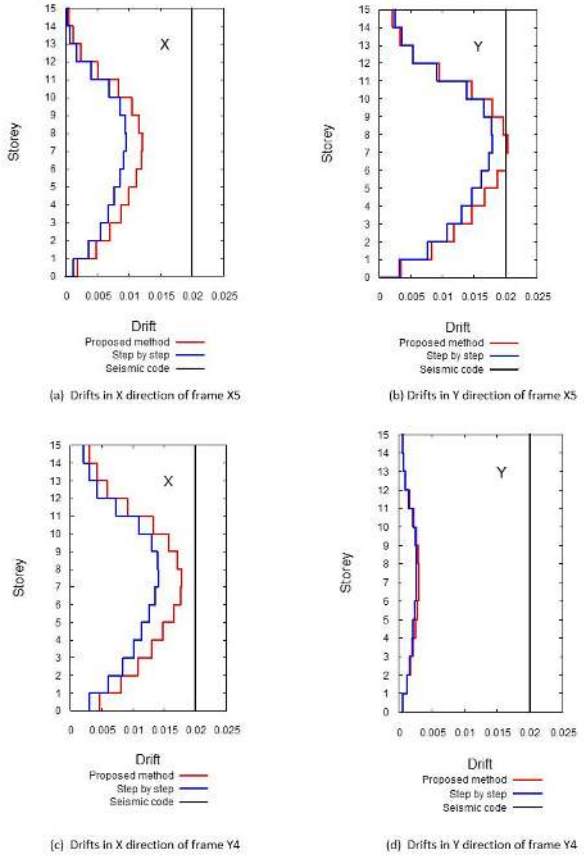


Figure 12 Comparison of the interstory drifts for15-story building.

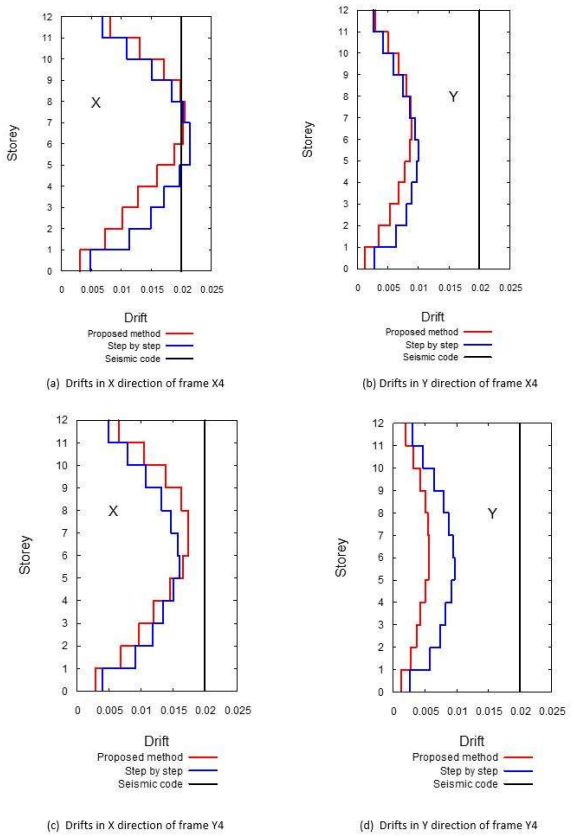


Figure 13 Comparison of the interstory drifts for12-story building.

6 CONCLUSIONS

This paper presents a procedure for the displacement-based seismic design of in-plan asymmetric buildings considering nonlinear behavior and control of structural damage. From the results of the design examples presented in this paper, which included an analysis of the performance results of the structures designed using the displacement-based seismic design method proposed, compared with those calculated using nonlinear dynamic step-by-step analysis of the buildings subjected to the same design seismic demand for which they were designed, the following conclusions may be derived:

- The original design method proposed for application to plane frames and modified in this investigation leads to performance results that are congruent with those obtained from a nonlinear dynamic step-by-step analysis. This method ensures that the expected design performance, *i.e.,* maximum interstory drift, using a reference system associated with the fundamental mode, is consistent, under certain circumstances mainly associated with the influence of higher modes on structural performance, with that calculated with a method that is numerically "exact".
- The behavior curve of the reference of a structure, derived from it and corresponding to the fundamental mode, has structural properties that are closely related to the seismic performance.
- A good control of structural damage was achieved. The differences between the distributions of structural damage assumed as a design objective in the application of this method and those obtained from nonlinear dynamic step-by-step analysis under the same seismic demand for which it was designed are not significant.

6 Acknowledgments

The authors acknowledge the Institute of Engineering, UNAM, and the National Council for Science and Technology, CONACyT, for the scholarship of the student author during his PhD studies and for the sponsorship of the project No 221526 "Development and Validation of a New Approach for the Seismic Evaluation and Displacement-based Design of Structures Considering Damage Control" on which this paper is based. The authors also acknowledge the University of Guanajuato Campus Celaya-Salvatierra for the financial support for the publication of this paper.

Author's Contribuitions: Conceptualization, O Palacios; Investigation, O Palacios and AG Ayala; Writing -original draft, O Palacios; Writing - review & editing, O Palacios, AG Ayala; Funding acquisition, O Palacios and AG Ayala, Resources, O Palacios; Supervision, O Palacios and AG Ayala.

Editor: Pablo Andrés Muñoz Rojas.

References

Beyer, K. (2007). Seismic design of torsionally eccentric buildings with RC U-shaped walls. Ph.D. Dissertation, Rose School, IUSS, Pavia, Italy.

Castillo, R. (2004). Seismic Design of Ductile Asymmetric Structures. Ph.D. Dissertation, University of Canterbury, New Zealand.

CDMX (2017). Building Code for the Mexico City and Complementary Technical Standards. RCCDMX-NTC.

CSI (2011). Perform 3D V5 Nonlinear Analysis and Performance Assessment for 3D Structures. Computers and Structures, Inc.: Berkeley, CA.

CSI (2009). SAP2000 Displacement-based design of reinforced concrete structures, Three-dimensional static and dynamic finite element analysis and design of structures. Computers and Structures, Inc.: Berkeley, CA.

Fakhraddini, A., Saffari, H. and Fadaee, M.J. (2019). Peak displacement patterns for the performance-based seismic design of steel eccentrically braced frames. Earthq Eng & Eng Vib, 18: 379-393

Fox, M.J., Beyer, K. and Sullivan, T.J. (2016). Predicting torsional response via an effective stiffness-based seismic assessment procedure. NZSEE Conference.

Georgoussis, G.K. (2013). Yield displacement profiles of asymmetric structures for optimum torsional response. Structural Engineering and Mechanics, 45(2): 233-257

Georgoussis, G.K. (2015). Minimizing the torsional response of inelastic multistory buildings with simple eccentricity. Canadian Journal of Civil Engineering, 42(11): 966-969

Gokdemir, H., Ozbasaran, H., Dogan, M., Unluoglu, E. and Albayrak, U. (2013). Effects of torsional irregularity to structures during earthquakes. Engineering Failure Analysis, 35: 713-717

Hong, H.P. (2013). Torsional responses under bidirectional seismic excitations: Effect of instantaneous load eccentricities. Journal of Structural Engineering, 139(1): 133-143

Lopez, S.E. (2009). Displacement-based design of reinforced concrete structures. Master of Engineering Thesis (Structures), UNAM, Mexico City.

Lucchini, A., Monti, G. and Kunnath, S. (2011). Nonlinear response of two-way asymmetric single-story building under biaxial excitation. Journal of Structural Engineering, 137(1): 34-40

Lumantarna, E., Lam, N. and Wilson, J. (2011). Simple seismic design and assessment of buildings incorporating the displacement controlled phenomenon. Australian Earthquake Engineering Society 2011 Conference.

Lumantarna, E., Lam, N. and Wilson, J. (2013). Displacement-controlled behavior of asymmetrical single-story building models. Journal of Earthquake Engineering, 17: 902–917

Miranda, B., MacRae, G.A., Yeow, T.Z. and Beyer, K. (2012). Torsional Considerations in Building Seismic Design. NZSEE Conference.

Paulay, T. (2002). An estimation of displacement limits for ductile systems. Earthquake Eng Struct Dynam, 31: 583-599

Paz, M., Leigh, W, (2004). Structural Dynamics, Kluwer Academic Publishers (USA).

Priestley, M.J.N. and Kowalsky, M.J. (2000). Direct displacement-based design of concrete buildings. Bulletin of the NZSEE, 33(4): 421-444

Wakchaure M.R. and Nagare, Y.U. (2013). Effect of torsion consideration in analysis of multi storey frame. International Journal of Engineering Research and Applications, 3(4): 1828-1832

Notation

α ratio of post-yield to elastic stiffness.
β factor between the responses to these demands.
δ_{d1}, δ_{d2} design interstory drifts.
δ_{di1}, δ_{di2} design interstory drifts for i^{th} mode contributions.
δ_{i1}, δ_{i2} interstory drifts of i^{th} modal form.
Δ_{irx}, Δ_{iry} roof displacement of i^{th} mode in X and Y direction.
ϕ_{ix}, ϕ_{iy}, $\phi_{i\theta}$ modal forms.
Γ_{xi}, Γ_{yi} modal participation factors in x and y direction.
ω_i^2 modal frequency.
μ ductility.
D_{ix}, $\dot{D}_{ix}$, $\ddot{D}_{ix}$ modal displacements, velocities and accelerations in X direction of SDOF.
$\bar{D}_{i1}$ yield displacement.
$\bar{D}_{ix2}$ ultimate displacement.
$\bar{D}_{ix}$, $\bar{D}_{iy}$ maximum displacement of mode i^{th} in X and Y direction.
S_{ay} strength per unit mass at yield.
S_{au} ultimate strength per unit mass.
$u_{ix}(t)$, $u_{iy}(t)$, $u_{i\theta}(t)$ displacements for the building in the time.
$\ddot{u}_{gx}(t)$ and $\ddot{u}_{gy}(t)$ demands in X and Y direction.
v_i, $\dot{v}_i$, $\ddot{v}_i$ modal displacements, velocities and accelerations.
v_{ix}, v_{iy} modal displacements in x and y direction.
$2\zeta_i\omega_i$ modal damping.